Tony Gee
Consulting Engineers

MTRCL Shatin to Central Link Contract 1112

Hung Hom Station & Stabling Sidings

Commission of Inquiry

Extended Inquiry

Structural Engineering Expert Report

This page is intentionally left blank

Document Issue Record

Project:	MTRCL Contract SCL 1112, Hong Kong
Report Title:	Commission of Inquiry – Structural Engineering Expert Report
Client:	O'Melveny & Myers LLP
Document No:	01
Revision:	01
Status:	Final
Date:	18-10-2019
Filename:	COI - Structural Engineering Expert Report of Nick Southward - Re COI Part 2 & Verification Report - 18 Oct 2019

Rev	**Date**	**Description and Purpose of Issue**	**Prepared**
01	18-10-19	First Issue	Nick Southward

Issuing Office:	Tony Gee and Partners (Asia) Ltd, 6F Chinachem Century Tower 178 Gloucester Road, Wanchai, Hong Kong
Tel:	+852 2377 2765
Email:	hongkong@tonygee.com

Contents

Appendix

1. Introduction

Leighton Contactors (Asia) Ltd ("LCAL") is constructing Contract SCL 1112, Hung Hom Station and Stabling Sidings, which forms part of the new Shatin to Central Link ("SCL") railway being constructed for the Mass Transit Railway Corporation Ltd ("MTRCL").

In respect of the diaphragm walls ("D-Walls") and platform slabs at the Hung Hom Station, a Commission of Inquiry ("COI") was established to inquire into the facts and circumstances surrounding the steel reinforcement fixing works and any other works which raise concerns about public safety and to ascertain whether the works were executed in accordance with the Contract.

In January of 2019, the remit of the COI was further extended by Government to consider other areas of the Project, namely the North Approach Tunnels ("NAT"), the South Approach Tunnels ("SAT") and the Hung Hom Stabling Sidings ("HHS") and other process related elements of the construction work carried out by LCAL. Hearings for the extended scope of the COI commenced in June 2019.

In the interim, an investigation process has been undertaken from December 2018 to April of 2019 to open up certain parts of the slabs which were the subject of the first part of the COI (aka the "Original Inquiry"). The purpose was to: (i) verify the as constructed condition of certain areas of the slabs; and (ii) randomly test whether the steel reinforcement bars had been properly connected to the couplers embedded in the diaphragm wall.

As a result of these tests, in July 2019 the MTRC have issued the Final Verification Study Report on the As Constructed Conditions of the North Approach Tunnels, South Approach Tunnels and the Hung Hom Stabling Sidings ("Verification Report"),[1] which proposes "suitable measures" to be carried out to the as constructed works in those areas (the "Works").

2. Instructions

I have been retained by O'Melveny and Myers (Counsel for LCAL) to provide my expert opinion on the following three principal items and the list of pertinent issues associated with each (as listed below). I have set out below the relevant sections of my report that address these issues.

2.1. Coupler connections / coupler engagement

(a) With reference to Sections 4.5.1 and 4.5.2 of the Verification Report, what is the spare structural capacity of the as-built works in the NAT, SAT and HHS?

Please refer to sections 4.1 and 4.2 below

(b) With reference to Sections 4.5.1 and 4.5.2 of the Verification Report, should a strength reduction factor be applied at any locations where couplers were used in the NAT, SAT and HHS? If so, what strength reduction factor should be applied at each of these locations?

Please refer to sections 4.3 to 4.5 below

[1] [BB16/9952ff]

(c) What are your comments on the "Updated Design Assumptions Adopted for Structural Review of HHS" in Appendix B3 of the Verification Report?

Please refer to sections 4.6 below

(d) Are the proposed suitable measures in relation to the HHS in Appendix C of the Verification Report necessary to ensure that the as-built works are structurally safe?

Please refer to sections 4.7 below

(e) What works (if any) should be performed to address any structural or other concerns arising from the coupler connections at locations in the NAT, SAT and HHS?

Please refer to sections 4.8 below

2.2. Shear link reinforcement and partial utilisation of shear

(a) With reference to Sections 4.5.3 and 4.5.4 of the Verification Report, should a strength reduction factor be applied at any locations in the NAT, SAT and HHS as a result of the shear links? If so, what strength reduction factor should be applied at each of these locations?

Please refer to Sections 5.1 to 5.3 below

(b) Are the proposed suitable measures in relation to the SAT in Appendix C of the Verification Report necessary to ensure that the as-built works are structurally safe?

Please refer to Sections 5.5 below

(c) What works (if any) should be performed to address any structural or other concerns arising from the shear links at locations in the NAT, SAT and HHS?

Please refer to Sections 5.6 below

2.3. COI's Directions

On 12 October 2019, and after the preparation of these list of pertinent issues in August 2019 (Appendix A to this report), it is my understanding that the COI have issued the following directions to the structural engineering experts:[2]

(a) The experts should focus on whether the as constructed works are safe and fit for purpose from a structural engineering perspective; and only if they are considered not safe or fit for purpose that such experts should then provide their opinion on whether the suitable measures are necessary for safety from a structural engineering perspective; and

(b) The experts shall not be required to look into the question of whether the suitable measures are required for statutory or code compliance.

[2] [AA2/472] [AA2/474].

3. Expert's declaration

I understand that my primary duty in preparing this report and giving evidence is to the COI, rather than to the party who engaged me and I have complied with that duty.

I have endeavoured in this report and in my opinions to be accurate and to have covered all relevant issues concerning the matters stated which I have been asked to address.

I have endeavoured to include in my report those matters, which I have knowledge of or which I have been made aware, that might adversely affect the validity of my opinion.

I have indicated the sources of all information that I have used.

I have not, without forming an independent view, included or excluded anything which has been suggested to me by others (in particular my instructing solicitors).

I understand that:

- My report, subject to any corrections before swearing as to its correctness, will form the evidence to be given under oath or affirmation.
- I may be cross examined on my report by a cross examiner assisted by an expert.
- I am likely to be the subject of public adverse criticism if the COI concludes that I have not taken reasonable care in trying to meet the standards set out above.

I believe the facts I have stated in this report are true and that the opinions I have expressed are correct.

4. Coupler connections / coupler engagement

4.1. Spare structural capacity of the as-built works in the NAT, SAT and HHS

I understand that there is no concern over the structural capacity at critical coupler locations in the NAT and SAT structures and therefore no suitable measures are proposed in these areas.[3]

However, it is alleged by MTRCL that the spare structural capacity of the trough wall kickers in the HHS area is not sufficient when a strength reduction factor of 35% (being the strength reduction factor adopted by MTRCL in the Verification Report) is applied to the coupled connections of the reinforcement in the walls.[4]

4.2. Trough walls

The trough walls are located at each side of the train tracks in the HHS area. Their principal purpose is to contain the trains in the case of a derailment, so that any adjacent structures are not at risk of damage.

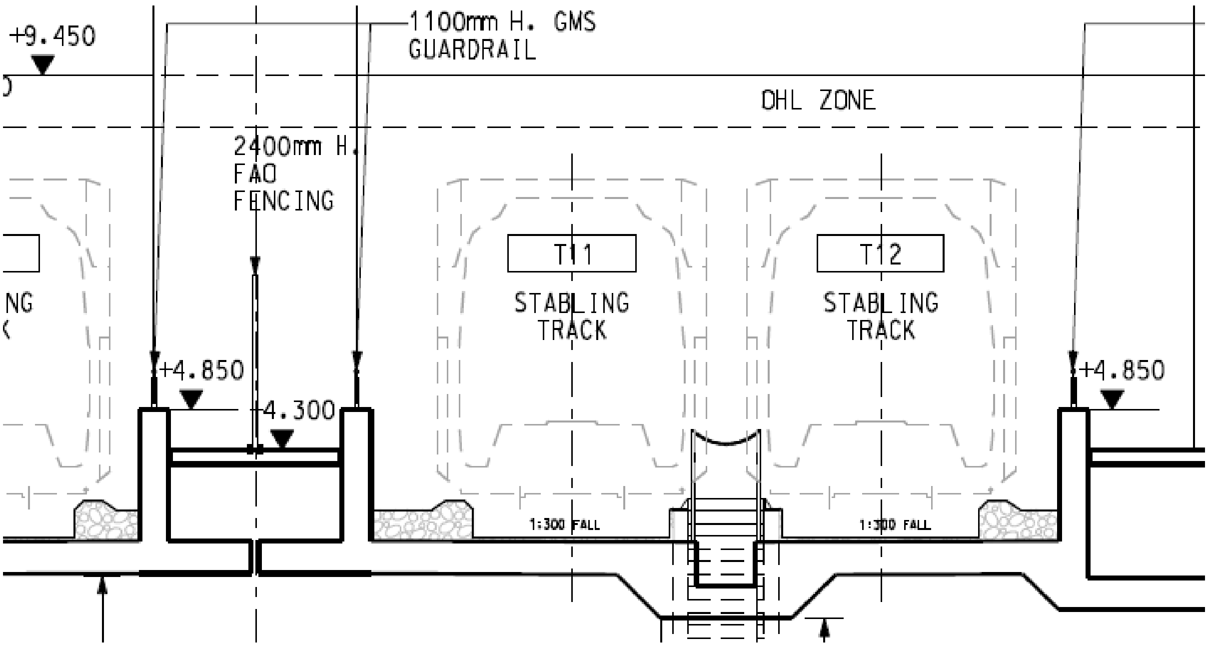

Figure 1 – Typical Arrangement of Trough Walls in HHS Area.

The walls were designed to resist a transverse horizontal impact load of 400 kN, applied at the top of the walls over a length of 2.2m. This is a loading that AECOM explain in Section 1.2.3 of their calculations[5] for the walls that has been agreed with the MTRCL Operations Division that is appropriate for the speed of the trains operating in the HHS area.

[3] See Section 4.5.1 of the Verification Report [BB16/9978].
[4] See Section 4.5.2 of the Verification Report [BB16/9978].
[5] See [DD18493].

4.3. What is the issue?

The walls were designed with a lapped connection between the starter bars that came up from the base slab and the continuation bars that extend to the tops of the walls.

LCAL replaced the lapped connections with BOSA bar couplers at some discrete locations for expediency of construction and to support site logistics, as shown in Figure 2 below.

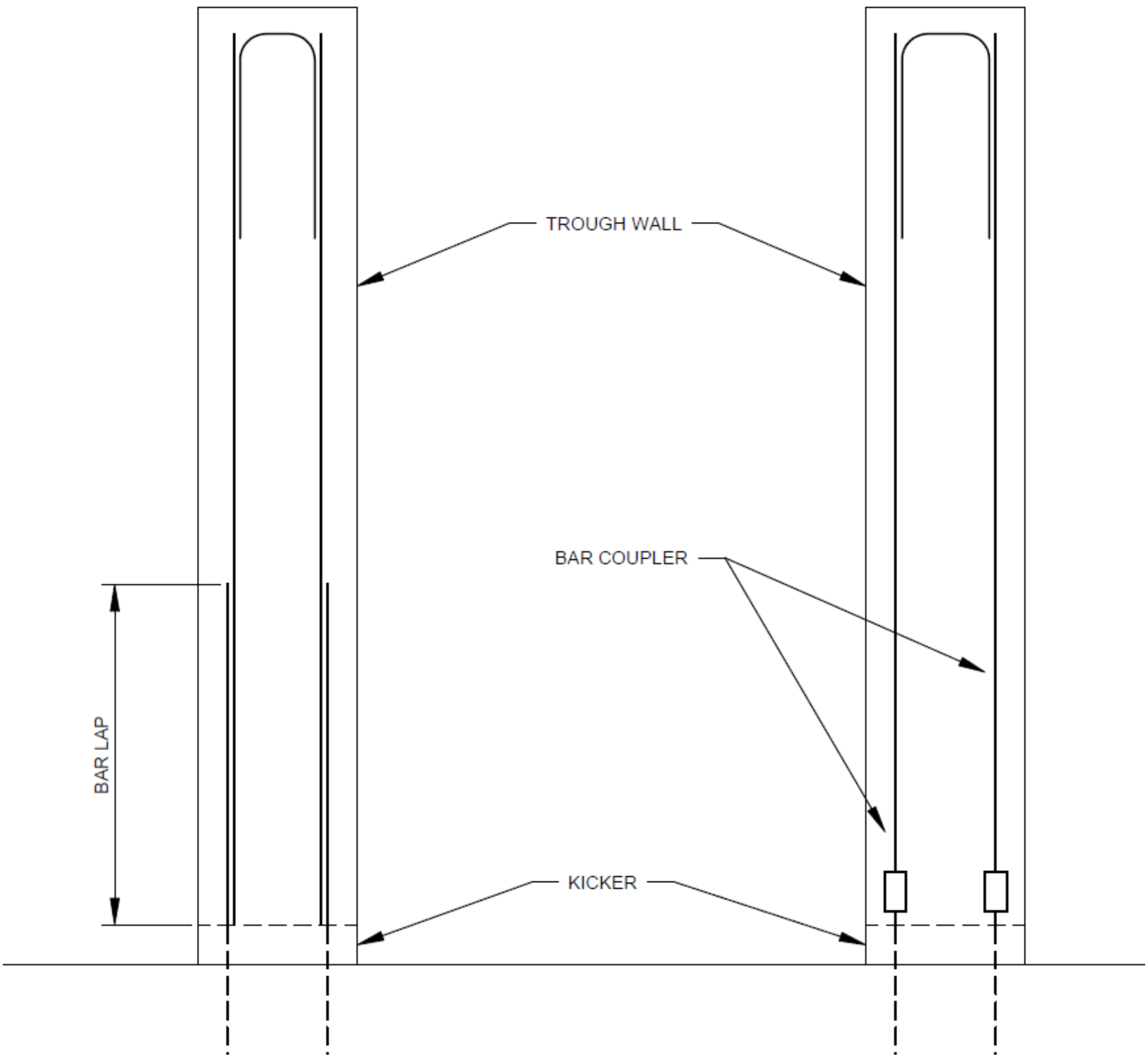

Figure 2 – Change to Trough Wall upstand

Figure 3 below shows a photograph obtained from LCAL of one of the trough walls while it was under construction. It is possible to see the bar couplers located just above the top of the kicker[6] for the wall.

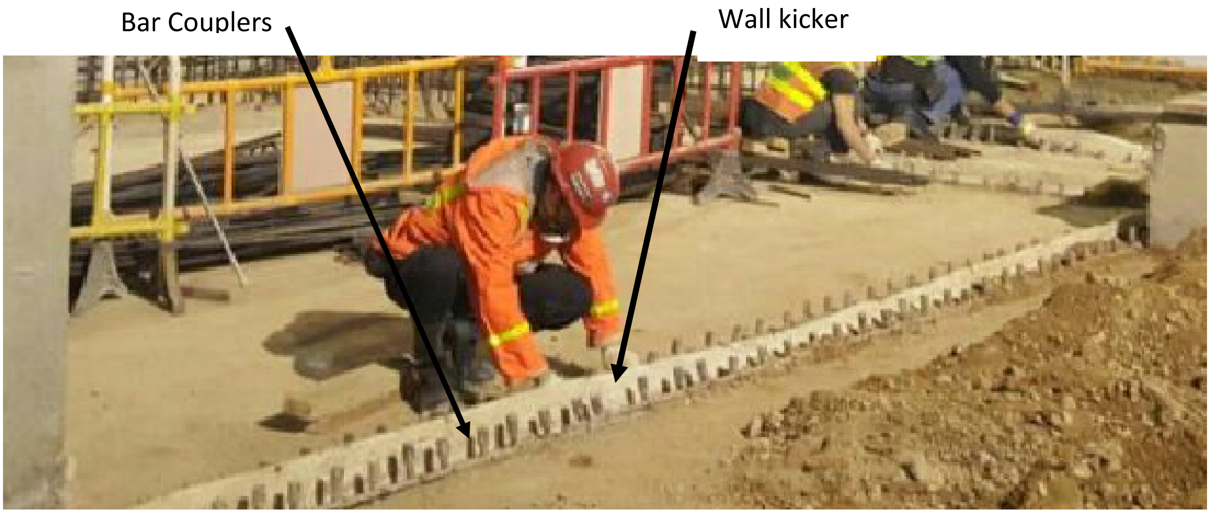

Figure 3 – Typical load spreading assumption of AECOM

4.4. AECOM's calculations

AECOM have carried out design calculations to check the structural capacity of the trough wall. These calculations are presented in two documents as follows:

- A calculation document for all areas of the trough wall kickers and the slabs that do not require any suitable measures [DD18/18482-18732].
- A calculation document to demonstrate the design of the proposed suitable measures for those areas of the trough wall kickers that (without the suitable measures) become unsatisfactory when a strength reduction factor of 35% is applied to the coupled connections [DD18/18733-19087].

There are no calculations which show the original capacity of the walls in the areas where the suitable measures are proposed by MTRCL (i.e. before those suitable measures are constructed). Both sets of calculations are not easy to follow, but I have ascertained that the areas where the suitable measures are proposed by MTRCL are at the expansion joints between the walls.

The train derailment / collision load case is one that is only considered at the Ultimate Limit State ("ULS") and this is confirmed in the AECOM calculations, section 1.2.4 [DD18/18493].

[6] A kicker is the bottom 100mm of wall which is concreted at the same time as the slab below the wall. The kicker forms the start of the wall and is used for securing the wall formwork.

4.4.1. Spread of load

Typically, AECOM's calculation approach is that when the impact load hits the wall, it is spread longitudinally on both sides of the impact point and the strength of the wall is mobilised in both directions, as shown in Figure 4 below.

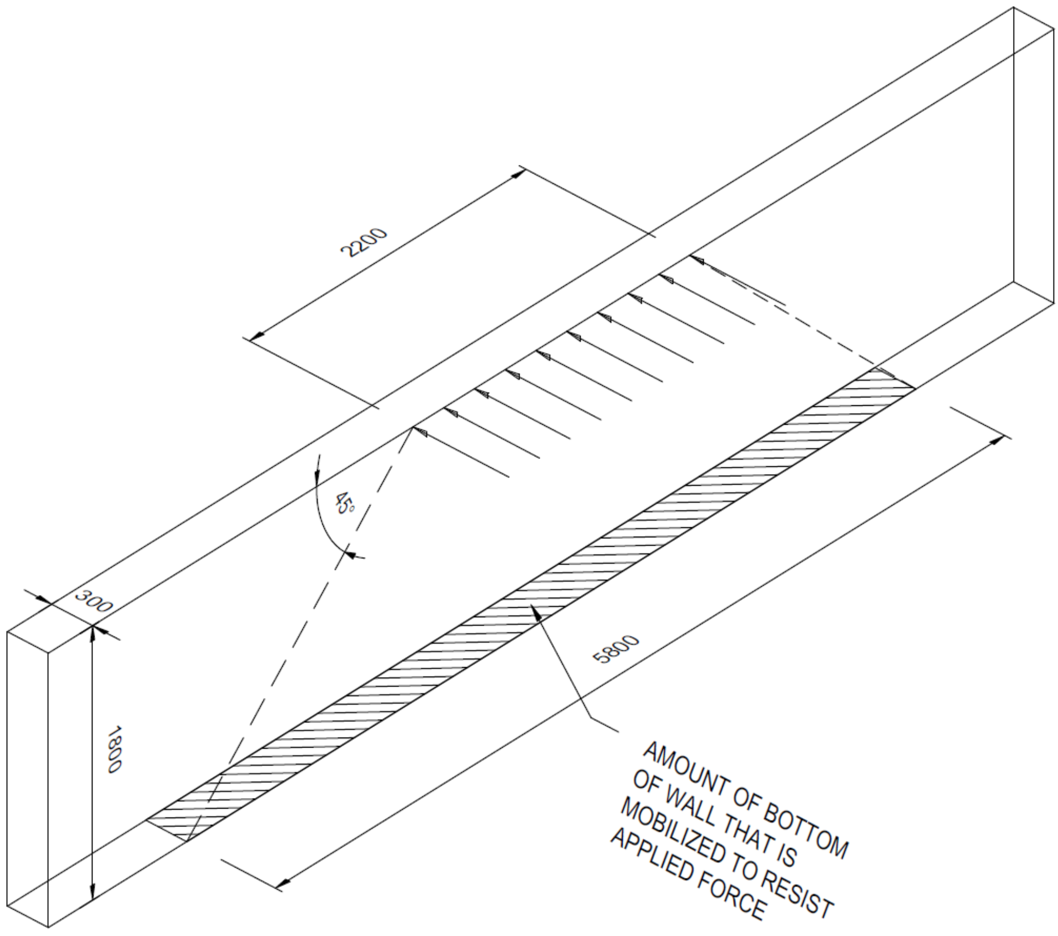

Figure 4 – Typical load spreading assumption of AECOM for 1.8m high wall

However, if the train were to hit the wall exactly at the location of an expansion joint (a small vertical gap between continuous sections of the wall) and such impact was so arranged that the start or end of the train impact load coincided exactly with the gap, then the load would only be able to be spread in one longitudinal direction, as the gap would prevent spread in the other direction, as shown in Figure 5 below.

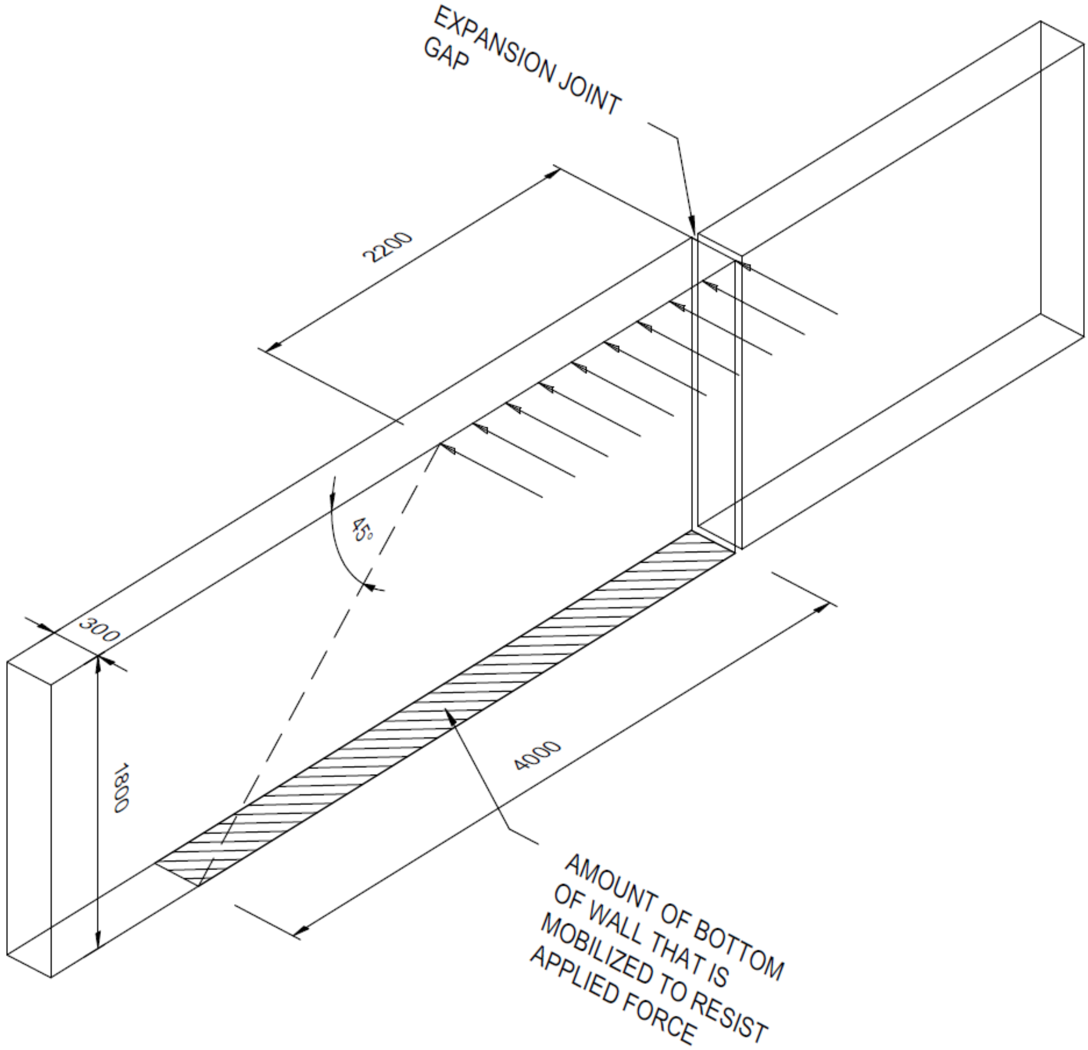

Figure 5 – Load spreading assumption of AECOM at expansion joints for 1.8m high wall

4.4.2. Load capacity

The AECOM calculations appear to show that there are 3 scenarios where the trough walls require suitable measures at expansion joint locations. The table below summarises these locations and contains the result of my own calculation for the ULS bending moments at the base of the wall, as well as the ULS wall capacities based on the full ULS strength of the reinforcement, as well as the percentage space capacity of the original design.

Wall Height mm	Wall Thickness mm	Wall Reinforcement	Design ULS Bending Moment kNm / 1.05m	Design ULS Capacity kNm kNm / 1.05m	Percentage spare capacity
2050	300	T25 @ 150	253	282.0	11%
2460	400	T20 @ 150	277	282.4	2%
1800	400	T20 @ 150	236	282.4	19%

My calculations are carried out on the same basis as that of AECOM, i.e. considering a 45 degree load spread in the vertically spanning wall, as shown in Figure 5 above, and are based on singularly reinforced concrete section capacity.

Based on my calculations, it can be seen that there is not 35% spare capacity in the ULS capacity. It follows that a 35% strength reduction factor of the coupled connections cannot be accommodated.

However, for the reasons explained below, I do not believe that a 35% strength reduction factor should be applied.

4.5. Should a strength reduction factor be applied to the couplers?

4.5.1. Comments on the statistical analysis

It is my understanding that no physical investigation works have been carried out in the HHS area and that the 35% reduction factor has been adopted by MTRCL based on statistical analysis carried out by MTRCL and Professor Yin for the purposes of the Holistic Report. In particular, it appears that the strength reduction factor of 35% has been adopted because it is comparable to the strength reduction factor applied in respect of the NSL slab.[7]

My comments on the strength reduction factor applied in respect of the NSL slab (and EWL slab) in the Holistic Report are set out in Section 6.10.2 of my second report for the Original Inquiry dated 11 October 2019.[8] As explained, I do not believe that the threshold for any binomial analysis conducted should have been set at an engagement length of 37mm. I would proceed in making any engineering judgement on the tests results in Appendix B3 of the Holistic Report by setting a threshold of 28mm for the embedded length and then using the figures derived from a statistical analysis of the coupler connections that satisfy this threshold.

[7] Section 4.2.6 of the Verification Report states: "*In the absence of any other alternative evidence or data, a strength reduction factor of 35% has been adopted. This is comparable to the strength reduction factor applied in respect of the NSL platform slab in the adjacent HUH Extension which is adjoining to NSL tunnel at SAT.*" [BB16/9976].

[8] See my second report for the Original Inquiry dated 11 October 2019, Item 14.1 of Bundle ER1.

Based on the threshold of 28mm, the defective rates significantly reduce from those reported in the Holistic Report to 16.3% (for EWL), 6.9% (for NSL) and 10.2% (for combined EWL and NSL).[9] Dr Wells also carried out the same analysis using the engagement length of 28mm and by adopting the "Missing Values Approach", which further reduced the defective rates to 14.5% (for EWL), 6.5% (for NSL) and 9.4% (for combined EWL and NSL).[10]

It is therefore my opinion that, if the strength reduction factor applied to the HHS area is to be determined by reference to the defective rate for the NSL slab, the correct figures should be 6.9% or 6.5% depending on whether a "Missing Values Approach" is adopted. This percentage is far less than the spare capacity demonstrated in section 4.4.2 above.

However, as explained below, I do not believe that there is enough similarity between the two areas for the same strength reduction factor to be applied in the HHS.

4.5.2. Difference in location and type of couplers

It is important to be aware that the location and arrangement of the couplers in the trough wall kickers is completely different from those at the junction of the D-Walls and EWL/NSL slabs in Areas A, B and C of the main station structure.

The couplers used in the junction of the D-Walls and EWL/NSL slabs were all for horizontal T40 bars, which were all embedded inside the D-Walls. During installation of the continuation bars, the only element of the coupler that was visible to the steel fixers was the end face. As a consequence, it would not have been possible to visually determine the alignment of the bar prior to its installation. This fact combined with the weight of the T40 continuation bars meant that it was difficult to hold, align and screw the bars into the couplers.

The situation of the couplers used for the trough wall kickers is completely different for the following reasons:

- The couplers were not embedded in the concrete and were fully visible, as shown in the photograph at Figure 3 above.
- The alignment of the coupler was therefore easily determinable by the steel fixers.
- The weight of the T40 continuation bars in the EWL / NSL slabs was either 20 kg or 40 kg depending upon the bar length.
- The continuation bars for the trough wall kickers were smaller (at either T20 or T25 size). The maximum weight of any continuation bar would therefore have been no more than 5.8 kg for the T20 bars and 7.9 kg for the T25 bars.
- As such, the weight of the continuation bars used for the trough wall kickers were so much lighter and therefore far easier to install and screw into the couplers.
- It would therefore have been considerably easier to screw the continuation bars into the couplers.

I do not believe therefore that there is enough similarity between the two situations for the results of the opening up investigation of the EWL and NSL slabs to be used in assessing the

[9] Paragraph 4.27 and Table 1 of the Expert Report of Dr Wells for the Original Inquiry.
[10] Ibid.

coupler connections at the HHS area. It follows that the defective rates and strength reduction factors applied to the EWL and NSL slabs cannot simply be adopted for the HHS area.

4.6. Updated design assumptions

I do not have any particular comment on the updated design assumptions proposed in Appendix B3 of the Verification Report. I believe it is correct that the design assumptions are updated for the structural review to reflect the as constructed conditions

However, I believe that consideration should also be given to the as constructed strength of the 28 day concrete. The strength of concrete used on construction sites is usually more than the design strength for the following reasons:

- So that the concrete supplier can be sure that the concrete supplied will meet this criteria (i.e. when the concrete samples are tested on site at 28 days old, strength tests will pass).
- The pressure of construction programmes often means that a high early age strength is needed so that the formwork can be stripped and the concrete becomes self-supporting as soon as possible after its pouring. If a concrete achieves a high early age strength, it normally means that the concrete will achieve a greater strength at 28 days than its required 28 day characteristic strength.

On this basis, it is likely that the strength of the concrete in the trough wall upstands is greater than that intended by the original design. This should be taken into account as part of any structural review of the relevant Works.

4.7. [redacted]

4.7.1. Effect of the train load being an ultimate load case

The MTRCL have specified that the train collision load case is to be considered at the ULS only. Therefore, in my opinion, the only property of a partially engaged coupler assembly that is relevant is that it should carry the static load capacity required by the HK Code of Practice for the Structural Use of Concrete 2004 version ("HKCOP") and Buildings Department.

Coupler assemblies are also required to prove cyclic tension and compression tests and perform so that the permanent deformation of the coupler is less than 0.1mm. These tests are relevant to the service performance of the coupler assemblies and not to a one off ultimate load event.

As discussed in my second report for the Original Inquiry dated 11 October 2019,[11] the ultimate strength of coupler assemblies with 6 or more threads engaged in the coupler has been proven via testing.

Therefore, if any strength reduction factor is required to be considered, it should be computed on the basis of 5 or less threads engaged in the coupler. It is my belief that this percentage will be less than the spare capacity demonstrated in section 4.4.2 above.

[11] See my second report for the Original Inquiry dated 11 October 2019, Item 14.1 of Bundle ER1.

4.7.2. Conservatism of AECOM analysis method

The method AECOM used to analyse the effect of the train collision load is conservative. They have designed on the basis that the wall is vertically spanning as a cantilever and that the amount of wall mobilised to resist the train force increases at an angle of 45 degrees down the wall.

I believe that they have used this 45 degree load distribution concept as it is simple, easy to understand, and in the belief that it would be more easily approved by the Buildings Department.

The problem in this case is that it is rather conservative, as it does not reflect the true behaviour of the wall.

4.7.3. Yield line analysis

As the train collision load is an ultimate load case, then use of a simplified elastic analysis to assess the effects of an ultimate load, although perfectly valid and accepted for use in design codes, is not the most accurate method.

The yield line theory is an ultimate load theory for slab design which better represents the behaviour of slabs in ultimate load conditions. The assumed collapse mechanisms are defined by a pattern of yield lines along which the reinforcement has yielded and the location of which depends upon the loading and boundary conditions. The theory is valid for under-reinforced[12] elements, which is the case in this instance as the walls are not heavily reinforced.

Use of yield line analysis is expressly permitted by the HKCOP, refer to section 5.1.2 *Methods of Analysis*.

Yield line analysis is codified in other international design codes, for example the American Association of State Highway and Transportation Officials ("AASHTO"). This organisation is the publisher of the AASHTO LRFD design code, which is used for the design of infrastructure in the United States of America and many other American influenced countries in the world. Refer to ASSHTO LRFD 2017 version Appendix A13, which is used for the design of bridge containment parapets. A bridge parapet fulfils the same function as the trough wall upstands, in that they resist collision loads.

Yield line analysis is also explained in detail in many civil engineering textbooks. For example, a common textbook used by civil engineering undergraduates is *Reinforced and Prestressed Concrete* by Kong and Evans.

4.7.4. Yield line analysis assessment of trough walls

I have therefore used yield line analysis to make a realistic assessment of strength of the trough wall upstands. Although the HKCOP allows use of yield line analysis, the method is not

[12] Reinforced concrete sections in which the steel reaches yield strain at loads lower than the load at which the concrete reaches failure strain are called under-reinforced sections.

spelled out in the document, so I have used the design rules within AASHTO LRFD 2007[13] version to ensure that a robust, appropriate and codified method is used.

The yield line for the situation where the impact is applied adjacent to a movement joint in the wall (the critical case) is shown in Figure 6 below. The wall has to be checked to prevent failure along this line and, if this can be demonstrated, then such failure will not occur.

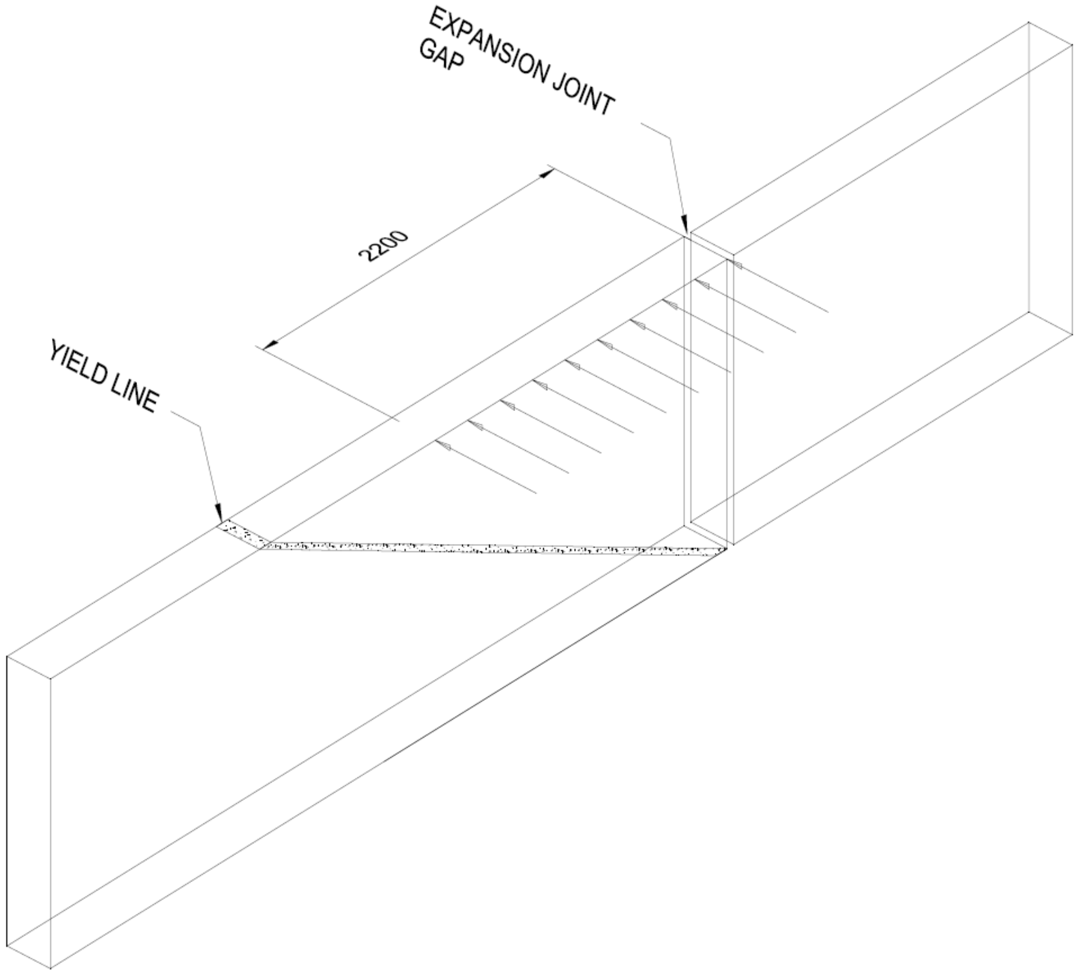

Figure 6 – Yield line approach at expansion joints

4.7.5. Initial yield line calculation

For this analysis, I have first followed the AECOM concept of reducing the stress in the vertical steel reinforcement at ULS to 299 N/mm² so that the 35% reduction in the strength of the coupled connections is included.

It is my understanding that a strength reduction factor of 4% has been applied to reinforcement that is 16mm or larger in diameter, refer to section 5.4 below. Although I

[13] 2007 version is used as the calculation formulae in this document are based on SI units whereas in the current 2017 version the formulae are based on Imperial units. The calculation is however the same in all other aspects.

disagree with this approach (for the reasons explained below), I have reduced the strength of the horizontal T16 steel by 4% in accordance with the MTRCL's strength reduction factor.

The full calculation is shown in Appendix B1 and is summarised as follows for the three different wall heights and thicknesses:

Result Summary of ADSEC analysis

Wall Ref.	Wall Thickness T mm	Wall Height H mm	Vertical Rebar Arrangement	Horizontal Rebar Arrangement	From ADSEC Moment Capacity in Vertical Direction, Muv kNm/1.05m	From ADSEC Moment Capacity in Horizontal Direction, Muh kNm/1.05m
Wall 1	300	2050	T25@150NF1	T16@150 NF2	195	113
Wall 2	400	2460	T20@150NF1	T16@150 NF2	188	171
Wall 3	400	1800	T20@150NF1	T16@150 NF2	188	171

Summary of Results of Yield Line Analysis

Wall Ref.	Mc Nmm/mm	Mw Nmm	Impact Load P=400*1.25 kN	Total Transverse Resistance Rw kN	UR= P/Rw	**Checking (if UR<1, then ok)**
Wall 1	185714	220619048	500	545	0.92	**OK**
Wall 2	179048	399457143	500	537	0.93	**OK**
Wall 3	179048	292285714	500	624	0.80	**OK**

This calculation shows that the there is a minimum spare capacity of 7% for the tallest wall of 2.46m (i.e. Wall 2), which is the most critical situation of the three walls.

4.7.6. Revised calculation

However, the calculation above is too conservative, because the vertical bending strength of the wall was reduced by 35% over its full height. In fact, the strength reduction of 35% should only be applied to the coupler assembly at the base of the wall. The rest of the wall has reinforcement that is only factored by the 4% strength reduction. Account must also be taken of the fact that the vertical steel reinforcement stops at the top of the trough wall and is lapped with a T16 U-bar over the top of the wall.

Therefore, the wall must be split into 3 vertical sections as follows:

- The top section of wall over the height of the lap between the main reinforcement and the T16 u-bar is taken with the bending strength of the T16 reinforcement, reduced by 4%.
- The lower section of the wall over the height of an anchorage length of the main wall reinforcement is taken with the bending strength of the main wall reinforcement, reduced by 35%.
- The remaining middle section of the wall is taken with the bending strength of the main wall reinforcement, reduced by 4%.

The yield line calculation is done using the weighted average of bending strength of these three sections of the wall along the horizontal length of the yield line.

Result Summary of ADSEC analysis

Wall Ref.	Wall Thickness T mm	Wall Height H mm	Vertical Rebar Arrangement	Horizontal Rebar Arrangement	From detailed calculation Average Weighted Moment Capacity in Vertical Direction, Muv kNm per 1.05m width	From ADSEC Moment Capacity in Horizontal Direction, Muh kNm per 1.05m width
Wall 1	300	2050	T25-150 Bottom+T16-150Top	T16-150	203	112
Wall 2	400	2460	T20-150 Bottom+T16-150Top	T16-150	229	169
Wall 3	400	1800	T20-150 Bottom+T16-150Top	T16-150	214	169

Summary of Results of Yield Line Analysis

Wall Ref.	Mc Nmm/mm	Mw Nmm	Lc mm	Impact Load P=400*1.25 kN	Total Transverse Resistance Rw kN	UR= P/Rw	Checking (if UR<1, then OK)
Wall 1	192887	218666667	2989	500	561	0.89	**OK**
Wall 2	218450	395942857	3490	500	618	0.81	**OK**
Wall 3	203495	289714286	3051	500	688	0.73	**OK**

This calculation shows that the there is a minimum spare capacity of 11% for the 300mm thick wall (i.e. Wall 1), which is the most critical situation of the three walls.

4.7.7. What strength reduction factor could be used for the couplers?

In light of the excess of capacity in the walls (as demonstrated above, even considering the 35% and 4% strength reduction factors proposed by MTRCL), I have considered the question of how much the strength reduction factor could be increased, but yet still ensure that the trough walls resist the train collision loading.

I have therefore re-done the calculation, but this time reducing the stress in the vertical steel in the wall until the structural capacity of the wall (calculated by yield line analysis) was reduced to equal the applied loading.

The full calculation is also shown in Appendix B and is summarised as follows for the three different wall heights and thicknesses:

Result Summary of ADSEC analysis

					Vertical rebar From previous calculation		Horizontal rebar From ADSEC
Wall Ref.	Wall Thickness T mm	Wall Height H mm	Vertical Rebar Arrangement	Horizontal Rebar Arrangement	Average Weighted Momen in Vertical Direction, Muv kNm per 1.05m width	**Reduced fy^ (for rebar in the lower region) Mpa**	Moment Capacity in Horizontal Directio kNm per 1.05m width
Wall 1	300	2050	T25-150 Bottom+T16-150Top	T16-150	177	**190**	112
Wall 2	400	2460	T20-150 Bottom+T16-150Top	T16-150	211	**190**	169
Wall 3	400	1800	T20-150 Bottom+T16-150Top	T16-150	189	**190**	169

Summary of Results of Yield Line Analysis

Wall Ref.	Mc Nmm/mm	Mw Nmm	Lc mm	Impact Load P=400*1.25 kN	Total Transverse Res Rw kN	UR= P/Rw	Checking (if UR<1, then ok)
Wall 1	168358	218666667	3068	500	504	0.99	OK
Wall 2	200957	395942857	3561	500	582	0.86	OK
Wall 3	179588	289714286	3128	500	624	0.80	OK

This calculation shows that the stress in the vertical steel can be reduced to 190 MPa for the 300mm thick wall (i.e. Wall 1) and the yield capacity of the walls is still more than the applied loading. It could be reduced even further for the other two walls.

190 MPa represents a 58% strength reduction in the coupler assembly. In other words, the MTRCL proposed strength reduction factor of 35% could actually be reduced further to 58%, and the walls would still be able to withstand the collision loads.

4.7.8. Conclusion

In conclusion, the above calculations demonstrate that the trough wall upstands can withstand the train collision loads, even when a 35% reduction factor is applied to the strength of the coupled connections.

In fact, the calculations also demonstrate that the coupled connections could be reduced in strength to 58% and the walls would still have sufficient load capacity.

4.8. What other works should be performed

As far as I understand, there are no other suitable measure proposed by the MTRCL in regards to the coupled connections.

4.9. Are the trough walls safe and fit for purpose?

I have demonstrated that the most critical areas of the trough walls are safe and can withstand the design loads with the as constructed coupler details, even allowing for a 35% strength reduction factor, i.e. assuming that 35% of the couplers are not fully engaged and are therefore discounted.

It follows that the trough walls are safe. They can withstand the design collision loads and have considerable reserve of strength.

5. Shear links

The Verification Report, in section 4.5.3 and 4.5.4, alleges that a strength reduction factor of 4% or 13% depending on the rebar size should be applied to the NAT, SAT and HHS structures.

In relation to the SAT structures, it is alleged that the shear links are deficient at certain areas of the SAT structures and suitable measures are required to enhance the shear strength. However, no suitable measures are proposed for the NAT and HHS in respect of shear links.

5.1. Atkins SAT shear assessment

No details of this deficiency were included in the Verification Report and neither were they included in the Assessment reports of Atkins that were provided to the COI in September 2019. Figure 7 below is extracted from the Atkins calculation report Vol. 1 [OU6/4026 - 4273], where it is advised that a separate report is relevant.

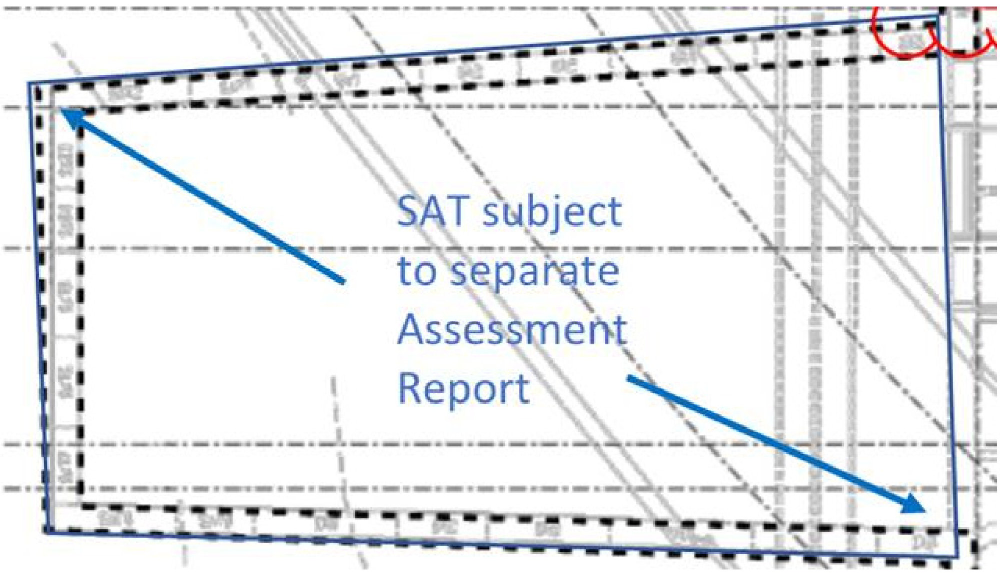

Figure 7 – Extract from Atkins Assessment Report

I received that "separate Assessment Report" [AA2/483ff] two days before the date of this report. I have therefore had limited time to review the assessment findings of Atkins.

5.2. Should the shear links be discarded in their entirety?

I understand from the calculations in Atkins' "separate Assessment Report" [AA2/483ff] that Atkins have completely discounted the presence of the as constructed shear links in the SAT slab in their current structural assessment.

I do not believe that it is appropriate to discard the presence of the as constructed shear links for the following reasons:

- The limited investigation measures of MTRCL do not prove that the shear links were not installed in the works in the SAT slabs.

- In Area A of the station structure, which is similar in slab thickness to that of the SAT structure, the only investigation done was by LCAL, which categorically proves the presence of the shear links.
- The concept of completely disregarding the as constructed shear links is analogous to the binomial approach adopted by MTRCL and Professor Yin in assessing the coupler connections in the D-Walls of the station structure. Dr Wells pointed out that adopting this approach would ascribe no loading capacity to coupler connections where the threads were still partially engaged.[14] In my report for the Original Inquiry, I explained that a partially engaged coupler with 6 or more threads engaged will contribute to the strength of the structure. Similarly, I am of the view that a shear link with partial placement contibutes to the strength of the structure. It is therefore wrong to completely disregard the shear links.

5.3. EIC shear calculation of SAT slab area

In my second report for the Original Inquiry dated 11 October 2019, I discussed the manner in which Atkins had performed their assessment of the shear capacities of the platform slabs of Areas A, B and C of the station structure.

5.3.1. Shear capacity

I pointed out that in their determination of v_c, the concrete shear stress capacity, several beneficial effects had not been considered that was adversely impacting on the representativeness of their calculations of the shear capacity of those slabs.

In a similar manner for the SAT slab, EIC have reviewed those Atkins calculations and corrected them to include the effect of:

- Correct tension steel area (in some cases this is less than Atkins used).
- Representative 28 day strength of the concrete which was supplied to site, which for this area is 55 MPa, after consideration of the standard deviation. Refer to Appendix C for a summary of the test results and standard deviation calculation, which has been prepared by EIC. [15]
- The redistribution of shear forces that would occur if a shear failure would occur in the NSL slabs as computed by Atkins.

5.3.2. Redistribution of shear force

If, as Atkins compute, a shear failure were to occur in the NSL slab at the SAT area, then in order for this to happen the hanger wall that connects the OTE and NSL together would be mobilised, so that the wall would then load the OTE slab above. EIC have calculated the excess shear force that needs to be re-distributed from NSL to OTE slab and checked that this is within the capacity of the hanger wall and the resulting shear force and bending moments imposed

[14] Day 4/67:14-68:5.

[15] I understand that LCAL will submit this analysis by EIC to MTRCL shortly and it will be disclosed to the COI in due course.

on the OTE slab can be carried by the OTE slab in addition to those shear and bending moments carried by the OTE slab that have been computed by Atkins. Figure 8 below, extracted from the EIC shear assessment calculations[16] (see Appendix D), shows the diagonal shear crack that would occur in the NSL slab and how it is resisted by the wall connection between the NSL and OTE slabs.

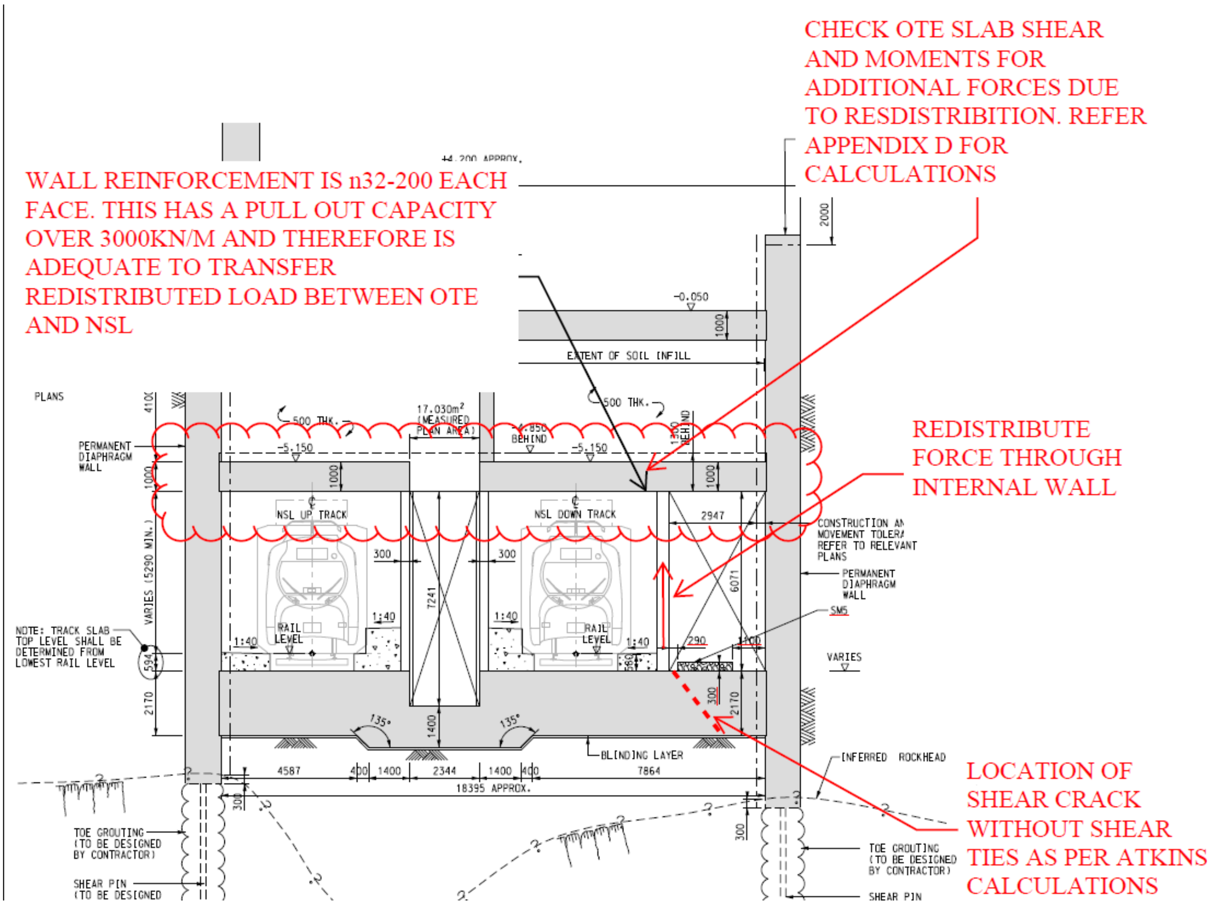

Figure 8 – Load spreading assumption of AECOM at expansion joints for 1.8m high wall

The modified utilisations are shown below, which have been calculated by EIC for all the areas in the slab where suitable measures have been proposed by Atkins, i.e. those areas where the shear capacity is less than the applied shear stress and minimum shear links are required.

The adjusted utilisations coloured in red after consideration of actual steel area and concrete strength (3rd coloured column from left to right) are then used to calculate the shear force that is required to be redistributed (coloured in yellow).

[16] I understand that LCAL will submit this analysis by EIC to MTRCL shortly and it will be disclosed to the COI in due course.

Reference Analysis Section	Reference joint	Current utilisation	v/vc (Actual Strength)	v/vc (Actual Steel)	v/vc (Actual concentrated load enhancement)	SHEAR FORCE TO BE REDISTRIBUTED, kn
NSL - Section 2A	25	1.071	0.964	0.998	No Change	
NSL - Section 2A	25	1.071	0.964	0.998	No Change	
NSL - Section 2A	25	1.102	0.991	1.026	No Change	-31.8935278
NSL - Section 2A	25	1.041	0.936	0.969	No Change	
NSL - Section 2A	25	1.031	0.927	0.960	No Change	
NSL - Section 2A	25	1.051	0.945	0.979	No Change	
NSL - Section 2A	25	1.041	0.936	0.969	No Change	
NSL - Section 2A	25	1.082	0.973	1.007	No Change	-8.586180068
NSL - Section 2A	25	1.061	0.955	0.988	No Change	
NSL - Section 2B	18	1.020	0.918	0.831	No Change	
NSL - Section 2B	18	1.114	1.002	0.774	No Change	
NSL - Section 2B	18	1.068	0.961	0.743	No Change	
NSL - Section 2B	18	1.000	0.900	0.815	No Change	
NSL - Section 2B	18	1.041	0.936	0.848	No Change	
NSL - Section 2B	18	1.125	1.012	0.782	No Change	
NSL - Section 2B	17	1.200	1.080	1.073	OK	-125.8323006
NSL - Section 2B	17	1.013	0.912	0.906	No Change	
NSL - Section 2B	17	1.173	1.056	1.050	OK	-84.91208152
NSL - Section 2B	17	1.267	1.139	1.133	OK	-226.8048454
NSL - Section 2B	17	1.160	1.044	1.038	OK	-64.43428788
NSL - Section 2B	17	1.213	1.092	1.085	OK	-145.6022561
NSL - Section 2B	17	1.067	0.960	0.954	OK	
NSL - Section 2B	17	1.133	1.020	1.014	OK	-23.61340194
NSL - Section 2B	17	1.187	1.068	1.062	OK	-104.8445104
NSL - Section 2B	17	1.213	1.092	1.085	OK	-146.8608385
NSL - Section 2B	17	1.267	1.139	1.133	OK	-228.2143007

The OTE slab shear and moment capacity ratios were then re-calculated, using the original Atkins capacity ratios and the load effects from the re-distributed shear force, as shown below and all areas of the slab are proven to be strong enough to resist this re-distribution.

		R IN OTE FOR ULS CASES WHERE REDISTRIBUTIO					CHECK MOMENT IN OTE FOR ULS CASES WHERE REDISTRIBUTION REQUIRED								
uls LOAD COMBINATION	FCU	Shear force to be added due to resdistributrion	new V, KN	new v, Mpa	Shear utisation		current peak moment in OTE slab from atkins B.M.D	additional moment at peak	Total moment	ATEEL	GAMMA M	FY EFFECTIVE	X	MOMENT CAPACITY	Moment utilisation
1	40	-125.832	-983.832	0.982849	47.0%		3838	-196.955	4034.954905	15476	1.5	400	346.4776	5,254.17	76.80%
2	40														
3	40														
4	40														
5	40														
6	40														
7	40	-84.9121	-980.912	0.979932	48.0%		3838	-132.906	3970.905867	15476	1.5	400	346.4776	5,254.17	75.58%
8	40	-226.805	-993.805	0.992812	49.1%		3838	-354.999	4192.998889	15476	1.5	400	346.4776	5,254.17	79.80%
9	40														
10	40														
11	40	-64.4343	-986.434	0.985449	51.9%		3838	-100.854	3938.853668	15476	1.5	400	346.4776	5,254.17	74.97%
12	40	-145.602	-995.602	0.994608	44.2%		3838	-227.899	4065.899183	15476	1.5	400	346.4776	5,254.17	77.38%
13	40														
14	40														
15	40														
17	40														
19	40														
21	40														
23	40														
25	40														
27	40														
29	40														
31	40	-23.6134	-984.613	0.98363	52.0%		3838	-36.9601	3874.960107	15476	1.5	400	346.4776	5,254.17	73.75%
32	40	-104.845	-992.845	0.991853	45.5%		3838	-164.104	4002.104451	15476	1.5	400	346.4776	5,254.17	76.17%
33	40	-146.861	-977.861	0.976884	51.4%		3838	-229.869	4067.869139	15476	1.5	400	346.4776	5,254.17	77.42%
34	40	-228.214	-987.214	0.986228	47.4%		3838	-357.205	4195.204992	15476	1.5	400	346.4776	5,254.17	79.85%
35	40														
36	40														
37	40														
38	40														

Therefore, a shear failure could not occur in the NSL slab as predicted by Atkins, if the presence of the as constructed shear links were completely disregarded (which, as I have explained in 5.2 is not the correct approach).

5.4. Strength reduction factors

5.4.1. Rebar not tested after delivery to site

It is my understanding that the strength reduction factors of 4% and 13% referred to in Section 4.5.3 and 4.5.4 of the Verification Report have been adopted by MTRCL as a result of evidence before the COI that approximately 7% of rebar used in the Project was not tested by a HOKLAS laboratory after it was delivered to site.[17]

I understand that the figures of 4% and 13% have been determined by taking the maximum reduction in tensile strength of 55 failed samples of 110,000 samples tested by MTRCL's HOKLAS laboratory since 2010.[18]

5.4.2. Statistical Expert Evidence

LCAL's statistical expert, Dr Wells, has applied statistical analysis to the approach adopted by MTRCL in determining the strength reduction factors of 4% and 13%. I understand that Professor Yin did not undertake any statistical analysis in relation to the rebar testing.[19]

Dr Wells has noted that the purpose of the testing conducted by MTRCL's HOKLAS laboratory is to verify or confirm the manufacturers' testing.[20] I understand that all rebar used in the Project passed the manufacturers' testing.[21] It was only a small percentage (approximately 7%) that was not re-tested after it was delivered to site.[22] Dr Wells' opinion is that the level of confidence in this small percentage of rebar that was not re-tested exceeds the requirement under the relevant quality assurance standards.

Dr Wells also commented on the data used by MTRCL in determining the strength reduction factors.[23] In particular, the data was used from all tests conducted by MTRCL's HOKLAS laboratory (i.e. it was not confined to the tests conducted on the rebar of the manufacturers who actually supplied rebar for the Project) and there was no attempt to cross-reference the source of the 55 failed samples with those manufacturers that supplied rebar for the Project.

Dr Wells concluded that the rebar testing conducted for the Project satisfies the relevant quality assurance standards and that the level of confidence in the rebar that was not tested after it was delivered to site exceeds that which is required under the relevant standards.[24]

[17] See Section 3.1.10 of the Verification Report [BB16/9965].
[18] See Section 4.3.2 of the Verification Report [BB16/9977].
[19] Professor Yin on Day 5/131:11-19.
[20] See Paragraphs 3.11 - 3.13 of Dr Barrie Wells' Report for the Extended Inquiry, Item 2 of Bundle ER1.
[21] See Karl Speed's Witness Statement at paragraph 59 [CC6/3761]; Paragraphs 3.11 - 3.13 of Dr Barrie Wells' Report for the Extended Inquiry, Item 2 of Bundle ER1.
[22] See Karl Speed's Witness Statement at paragraph 8 [CC11/7288].
[23] See Paragraphs 3.16 of Dr Barrie Wells' Report for the Extended Inquiry, Item 2 of Bundle ER1
[24] See Paragraphs 3.14 of Dr Barrie Wells' Report for the Extended Inquiry, Item 2 of Bundle ER1

5.4.3. Conclusion

From a structural engineering perspective, I do not believe that it is appropriate to adopt any strength reduction factor because of the small percentage of rebar that was not re-tested after it was delivered on site. All of the evidence indicates that the rebar used in the Project was of satisfactory quality and that the re-testing conducted after the rebar was delivered to the site was sufficient to satisfy the relevant standards.

5.5. [redacted]

[redacted] The shear calculations of Atkins and EIC demonstrate that the shear demand is less than the shear capacity of the concrete and only the nominal minimum shear links are required in limited areas of the slabs.

As demonstrated in my second report for the Original Inquiry dated 11 October 2019, the demand on these minimum shear links is less than 100% because the shear links are over-provided at T12@300 centres (377mm²/m) compared to the requirement for minimum shear links of 300mm²/m.

Therefore, a reduction in effective anchorage length of the shear link is possible. In other words, the over-provision of the shear reinforcement is such that the as constructed links can still carry the ultimate loads and therefore the safety of the structure is ensured.

In my opinion, there is no justification for completely disregarding the shear links in the design calculations for the following reasons:

- The limited investigation measures of MTRCL do not prove that the shear links were not installed in the works.
- In Area A, where MTRCL require most of the suitable measures to be carried out, the only investigation undertaken in that area (i.e. by LCAL) categorically proves the presence of the shear links.
- The evidence of the as constructed shear links show that links were used that can carry the design loads due to their over-provision and are therefore compliant with the HKCOP.
- The detailing rule for shear links, in terms of the 10 x bar diameter distance required past the end of the bend, is applicable and required for shear links that carry the full ULS design load.
- The HKCOP allows modification of anchorage lengths dependent upon the design load in the bars.
- I have presented the justification for a reasonable extrapolation of the anchorage mechanism through which the shear link works in practice, which means that because of the over-provision of the shear links when compared to the minimum steel requirements, the straight length of the shear link can be reduced to 70mm without compromising the design strength of the structure.

As the shear links are HKCOP code compliant and are capable of taking the full loading, there is no justification for the proposed suitable measures of shear strength enhancement.

5.6. Are any other works required

As far as I understand, there are no other suitable measures proposed by the MTRCL in regards to the shear links.

5.7. Is the SAT Area safe and fit for purpose?

Yes. Only the shear strength of the SAT slabs has been questioned and for the reasons stated above these concerns are not valid. In my opinion, which is based on the structural assessments carried out, the entire structure is safe and fit for purpose.

6. Conclusion

[redacted]

[redacted] in my opinion [redacted]

- Tests have proven the partially engaged couplers can withstand the design loads.
- Yield line analysis of the trough walls in the HHS areas show that the as constructed walls, even with a strength reduction factor of 35%, can withstand the train collision loads and therefore are safe and fit for purpose.
- Due to the redundancy of the structure in the SAT area, the shear failure predicted by Atkins if the as constructed shear links were ignored could not occur.
- The shear links are over-provided and the as-constructed shape of the shear links can resist the applied design loads.